J. P. CARRARA & SONS, INC. | # 23446-014 | M. WEIGAND 2/11/15 | 1

FAIRFIELD VT BRIDGES

ELM BROOK ROAD BRIDGE

1 — 2-11-15 SOLID SLAB LIFT DESIGN

2 — 2-12-15 ADD'L EVALUATION

STATE OF VERMONT
ERNEST J. BROD
REGISTERED
PROFESSIONAL ENGINEER

PRECAST LIFTING DESIGN CALCULATIONS

(WORK WITH CARRARA SHOP DRAWINGS)

ELM BROOK SOLID SLAB BEAM:

WT OF SLAB = 27.73 TON

THERE ARE (4) LIFT POINTS AND ASSUMING A 60° SLING ANGLE W/ THE HORIZONTAL,

DESIGN LOAD @ EACH LIFT POINT

$$= \frac{27.73 \times 2}{4 \times 0.866} = 16.0^K$$

FROM ATTACHED DAYTON LITERATURE

USE 8 TON × 13 3/8" S.L. ANCHORS SWL(4:1 SF) = 16K

OK, WILL ACCEPT

DAYTON TABLE BASED ON $f'_c = 1.6ksi$

WHEREAS $f'_{ci} = 6.0ksi$

2 — 2-12-15 IN RESPONSE TO VT AOT COMMENTS REGARDING EDGE DISTANCE, JRC ENGINEERING JUDGMENT ACCEPTS S.L. ANCHOR WITH REDUCED EDGE DISTANCES.

EVALUATE LIFTERS AS HEADED STUDS AS PER PCI 6.5.3 & 6.5.4 (SECTION ATTACHED)

$\phi P_c = 10.7\, l_e(l_e + d_h)\lambda\sqrt{f'_c}$ WHERE $l_e = 13\frac{3}{8}"$ $d_h = 2\frac{5}{8}"$ $f'_{ci} = 6000psi$

$$= 10.7(13\tfrac{3}{8})(13\tfrac{3}{8}+2\tfrac{5}{8})(1.0)\sqrt{6000}\,/1000$$

$$= 177.4^K$$

→

TOPS

(CONTINUED)

FOR S.F. (4:1)

$SWL = 177.4/4 = 44.3K$

ACCOUNTING FOR $11\frac{3}{4}$" EDGE DISTANCE & 12" SPACING BETWEEN INSERTS:

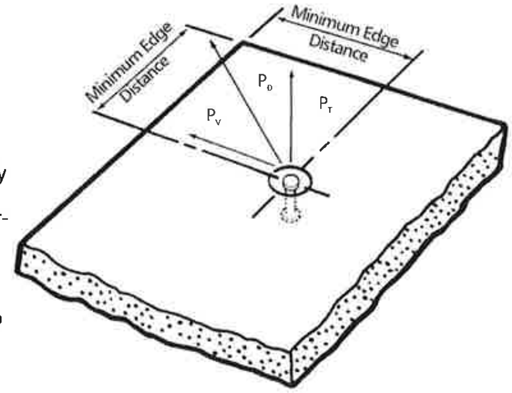
$$SWL = \frac{11\frac{3}{4}}{13\frac{3}{8}} \times \frac{12}{13\frac{3}{8}} \times 44.3 = 34.9K > 16.0K$$

O.K.

JDC WILL ACCEPT PLACEMENT OF S.L. ANCHORS AS SHOWN IN JDC SHOP DRAWINGS.

P-52 Swift Lift® Anchor Tensile and Shear Capacity

When anchors are used in the face of thin concrete elements



The following table lists the P-52 Swift Lift Anchors that are currently manufactured. Other sizes and lengths are available on special order. However, the sizes and lengths of anchors shown will handle the majority of flat precast concrete elements.

When the P-52 Swift Lift Anchor is properly embedded in normal weight concrete, the tabulated working loads are applicable for any direction of load. This applies even if the direction of load is parallel to the axis of the anchor, perpendicular to it or at any other angle.

Minimum distance between anchors is twice the minimum edge distance.

It is critical to remember that in order to obtain the safe working loads listed in the table below, the normal weight concrete must have obtained the minimum concrete strength shown, prior to initial load application.

Swift Lift Anchor Ton x Length	Safe Working Load	Minimum Concrete Strength	Minimum Edge Distance
1 ton x 2-5/8"	1,700 lbs.	3,500 psi	8"
1 ton x 3-3/8"	**2,000 lbs.**	**2,200 psi**	**10"**
1 ton x 4-3/8"	2,000 lbs.	1,600 psi	10"
1 ton x 8"	**2,000 lbs.**	**1,600 psi**	**10"**
1 ton x 9-1/2"	2,000 lbs.	1,600 psi	10"
2 ton x 2-3/4"	**2,100 lbs.**	**3,500 psi**	**8"**
2 ton x 3-3/8"	2,900 lbs.	3,500 psi	10"
2 ton x 5-1/2"	**4,000 lbs.**	**1,600 psi**	**13"**
2 ton x 6"	4,000 lbs.	1,600 psi	13"
2 ton x 6-3/4"	**4,000 lbs.**	**1,600 psi**	**13"**
2 ton x 11"	4,000 lbs.	1,600 psi	14"
4 ton x 3-3/4"	**4,000 lbs.**	**3,500 psi**	**12"**
4 ton x 4-1/4"	4,900 lbs.	3,500 psi	13"
4 ton x 4-3/4"	**5,800 lbs.**	**3,500 psi**	**14"**
4 ton x 5-1/2"	7,400 lbs.	3,500 psi	17"
4 ton x 5-3/4"	**7,900 lbs.**	**3,500 psi**	**17"**
4 ton x 7-1/8"	8,000 lbs.	1,800 psi	20"
4 ton x 9-1/2"	**8,000 lbs.**	**1,600 psi**	**17"**
4 ton x 14"	8,000 lbs.	1,600 psi	18"
4 ton x 19"	**8,000 lbs.**	**1,600 psi**	**20"**
8 ton x 4-3/4"	6,400 lbs.	3,500 psi	16"
8 ton x 6-3/4"	**11,200 lbs.**	**3,500 psi**	**21"**
8 ton x 10"	16,000 lbs.	3,500 psi	19"
8 ton x 13-3/8" ✓	**16,000 lbs.**	**1,600 psi**	**23"**
8 ton x 26-3/4"	16,000 lbs.	1,600 psi	27"
20 ton x 10"	**25,000 lbs.**	**3,500 psi**	**24"**
20 ton x 19-3/4"	40,000 lbs.	3,500 psi	31"

Safe Working Loads provide a factor of safety of approximately 4 to1 in normal weight concrete. Safe Working Load is based on anchor setback from face of concrete "X" dimension, as shown on page 26.

ability of headed stud design. The design methods used here should be considered an interim step toward a final headed stud design procedure. It is recommended that this procedure be limited to headed studs with an embedment not greater than 8 in.

An important factor in the performance of headed studs when controlled by concrete capacity is the confinement of the failure area with reinforcement. In shear, design capacity is increased with such reinforcement. In tension, ductility can be provided. It is recommended that reinforcement be placed to cross failure planes around headed stud anchorages.

Welded headed studs are designed to resist direct tension, shear or a combination of the two. The design equations given below are applicable to studs which are welded to steel plates or other structural members, and embedded in unconfined concrete.

Where feasible, headed stud connections should be designed and detailed such that the connection failure is precipitated by failure (typically defined as yielding) of the stud material rather than failure of the surrounding concrete. The in-place strength should be taken as the smaller of the values based on concrete and steel.

6.5.2.1 Tension

The design tensile strength governed by concrete failure is [9]:

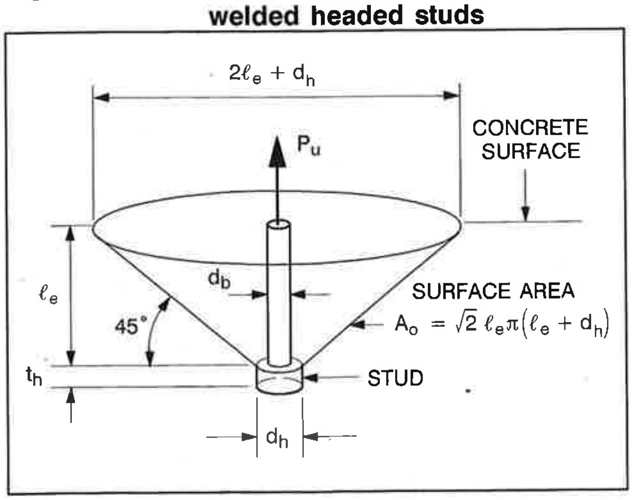
$$\phi P_c = \phi A_o(2.8\lambda\sqrt{f'_c}) \qquad \text{(Eq. 6.5.2)}$$

where:

ϕ = 0.85

A_o = area of the assumed failure surface which, for a single stud not located near a free edge, is taken to be that of a 45° truncated cone as shown in Figure 6.5.3.

Figure 6.5.3 Shear cone development for welded headed studs



Using the 45° cone area and ϕ = 0.85, Eq. 6.5.2 may be written as: 2A

$$\phi P_c = 10.7\ell_e(\ell_e + d_h)\lambda\sqrt{f'_c} \qquad \text{(Eq. 6.5.3)}$$

Note: The stud length is often used in place of the actual embedment length, ℓ_e, which is equal to the stud length minus the thickness of the head. This simplification is generally acceptable except in short studs. In short studs (length ≤ 4 in.), the use of actual embedment length is recommended. It should also be noted that short stud capacities are also sensitive to fabrication tolerances. Thus, use of a larger overall factor of safety may be appropriate for short studs. See Sect. 6.3.

For a stud located closer to a free edge than the embedment length, ℓ_e, the design tensile strength given by Eq. 6.5.3, should be reduced by multiplying it by C_{es}:

$$C_{es} = \frac{d_e}{\ell_e} \leq 1.0 \qquad \text{(Eq. 6.5.4)}$$

where d_e is the distance measured from the stud axis to the free edge. If a stud is located in the corner of a concrete member, Eq. 6.5.4 should be applied twice, once for each edge distance. Figure 6.15.6 lists values based on Eqs. 6.5.3 and 6.5.4.

For a group of studs, the concrete failure surface may be along a truncated pyramid rather than separate shear cones, as shown in Figure 6.5.4.

For this case, the design tensile strength is:

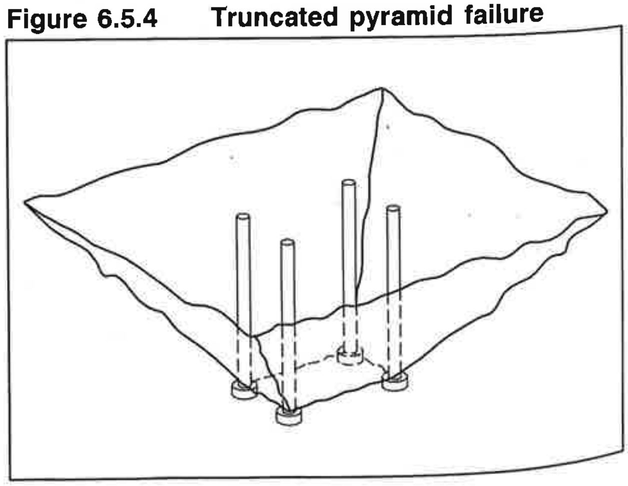
$$\phi P_c = \phi\lambda\left(\frac{2}{3}\right)\sqrt{f'_c}\,(2.8A_{slope} + 4A_{flat}) \qquad \text{(Eq. 6.5.5)}$$

where:

A_{slope} = sum of the areas of the sloping sides

A_{flat} = area of the flat bottom of the truncated pyramid

Figure 6.5.4 Truncated pyramid failure



ELM BROOK RD BRIDGE # 46 FA-AB3 ABUTMENT

$WT = 19.2^{T}$

THERE ARE (2) LIFTING LOOPS, AND THE MINIMUM SLING ANGLE W/THE HORIZONTAL IS 60°

$$\text{DESIGN LOAD/LIFT LOOP} = \frac{19.2 \times 2}{2 \times 0.866} = 22.2^{k}$$

FROM ATTACHED PCI LITERATURE, P 5.28

USE (9) 0.600" ϕ × 270 KSI STRANDS/LIFT LOOP

MIN EMBED ~~4'-10"~~ 4'-5"** SWL (4:1 S.F.) $\approx 1.1 \times \frac{(29+41)}{2}$ =

$38.5^{k} > 22.2^{k}$, OK

CHECK USING ATTACHED PCI LITERATURE, FIG 5.4.7A

$$\phi P_{c1} = \frac{0.85 \times 0.67 \sqrt{3500}\,(36")(58.75")}{1000} = 284^{k};\ 4'\text{-}10\tfrac{3}{4}"$$

LONG CHEEK WALLS.

FOR 4:1 S.F.

$SWL = 284/4 = 71^{k} > 22.2^{k}$, OK

SEE SHT 5 FOR LIFT LOOP LOCATION

△1 2-11-15

** REDUCE LIFT LOOP EMBEDMENT FROM 4'-10" TO 4'-5" TO MAINTAIN 3" CLEAR FROM BOTTOM OF ABUTMENT. EMBEMENT REDUCTION DOES NOT AFFECT CAPACITY OF LIFT LOOPS.

ELM BROOK RD BRIDGE #46 FA-AB4 ABUTMENT

$WT = 28.94^T$

THERE ARE (4) LIFTING LOOPS AND THE MINIMUM SLING ANGLE W/ THE HORIZONTAL IS 60°

DESIGN LOAD/LIFT LOOP $= \dfrac{28.94 \times 2}{2 \times \sin 60} = 33.4^k$

FROM ATTACHED PCI LITERATURE, 11" 5.28

USE 4, 0.600" φ x 270 KSI STRANDS/LIFT LOOP

MIN EMBED (~~6'-10"~~ 6'-5" **) SWL(4:1 S.F.) $= 1.1 \times \left(\dfrac{29+41}{2}\right)$

$= 38.5^k > 33.4^k$, O.K

CHECK USING ATTACHED PCI LITERATURE, FIG 6.15.7A

$\phi P_c = \dfrac{0.85 \times 2.67 \sqrt{3500}\,(36")(58.75")}{1000} = 284^k$; $4'\text{-}10\tfrac{3}{4}$ LONG CHECK WALL

FOR 4:1 S.F.

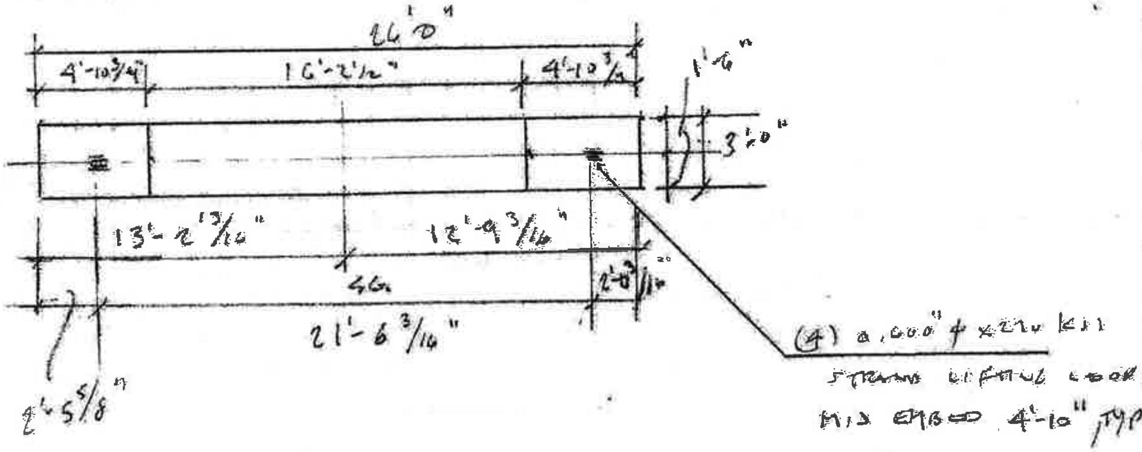
$SWL = 284/4 = 71^k > 33.4^k$, O.K

SEE SHT 5 FOR LIFT LOOP LOCATIONS

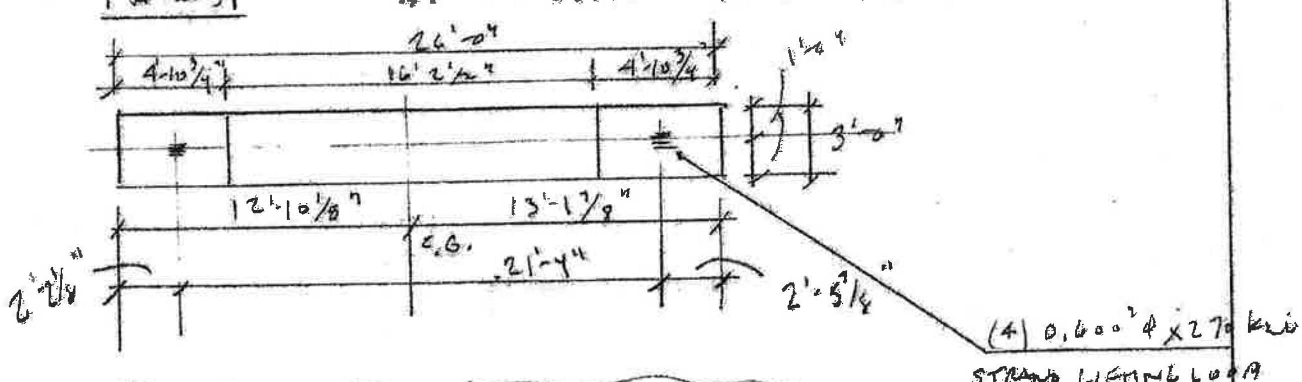
△1 2-11-15

** REDUCE LIFT LOOP EMBEDMENT FROM 6'-10" TO 6'-5" TO MAINTAIN 3" CLEAR FROM BOTTOM OF ABUTMENT. EMBEDMENT REDUCTION DOES NOT AFFECT CAPACITY OF LIFT LOOPS.

FA-AB3 WT 19.2^K PARTIAL PLAN VIEW

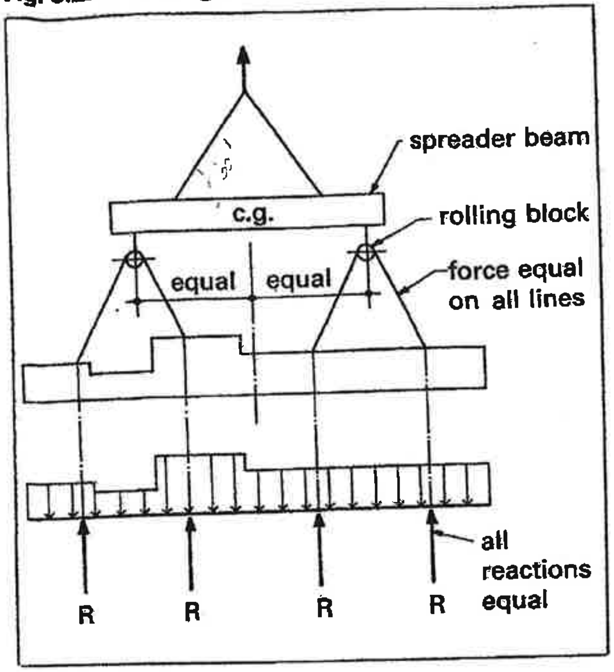


FA-AB4 WT 28.94^K PARTIAL PLAN VIEW



NOTE: LIFTING LOOP LOCATIONS MAY BE ADJUSTED SLIGHTLY TO FACILITATE FABRICATION

Fig. 5.2.10 Arrangement for equalizing lifting loads



lines equal. The member can then be analyzed as a beam with varying load supported by equal reactions.

The force in inclined lift lines can be determined from Fig. 5.2.7.

5.2.8 Handling devices

The most common lifting devices are prestressing strand or cable loops projecting from the concrete, threaded inserts, or special proprietary devices.

Since lifting devices are subject to dynamic loads, ductility of the material is part of the design requirement. Deformed reinforcing bars should not be used since the deformations result in stress concentrations from the shackle pin. Also, reinforcing bars are often hard-grade or re-rolled rail steel with little ductility and low impact strength at cold temperatures. Smooth bars of a known steel grade may be used if adequate embedment or mechanical anchorage is provided. The diameter must be such that localized failure will not occur by bearing on the shackle pin.

Prestressing strand is often used for lifting loops. The variables involved make it almost impossible to calculate a capacity which can be used for all situations. Generally, producers will establish standard criteria for use in handling the standard products manufactured by that plant. Table 5.2.3 is an example which has been used successfully.

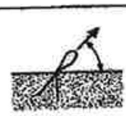
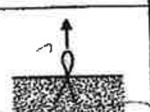
Reduced capacities for shorter embedment lengths may be suitable. In shallow products, providing a 90° bend can reduce the required embedment length significantly. Lightly rusted strand has better bond than bright strand.

The diameter of the bend of the loop should be at least 4 in. For smaller diameters, the loop capacities in Table 5.2.3 should be reduced to:

1 in. dia. — 70 %
2 in. dia. — 85 %
3 in. dia. — 90 %

The angle of incline of lifting has little effect on the strand lifting loop capacity if the angle from the horizontal is more than about 20°. Typical handling methods are usually such that this angle is no less than 60°.

Table 5.2.3 Capacity of ½ in. diameter, 270 ksi strands used as lifting loops

Lifting angle		Embedment length (in.)	Single loop (kips)	Double loop (kips)	Triple loop (kips)
45 degrees		16	5	8.5	11.5
		22	8	13	17.5
		28	10	18	23
		34	11	23	29 ✓
Vertical		16	7.5	12.5	16.5
		22	11.5	19	24.5
		28	15.5	25.5	33
		34	16	32.5	41 ✓

1. These values are limited by slippage rather than strand strength, with a factor of safety of 4. For other strand diameters, multiply table values by 0.75 for ⅜ in. diameter, 0.85 for 7/16 in. diameter, and 1.1 for 0.6 in.diameter.
2. Minimum f'_c = 3000 psi.
3. Multiple strand loops must be fabricated to ensure equal force on each strand.

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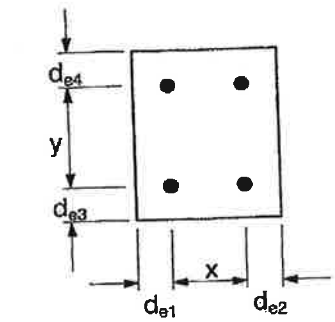
Figure 6.15.7A (continued) Design tensile strength for $h \geq h_{min}$, ϕP_{c1}—Case 6



x and y are the overall dimensions (width and length) of the stud group.

Case 6: Free edges on four adjacent sides

$\phi P_{c1} = \phi 2.67 \lambda \sqrt{f'_c}\,(x_1)(y_1)$

$\phi = 0.85$

where: x_1 and y_1 are the dimensions of the flat bottom of the part of the truncated pyramid.

For Case 6: $x_1 = x + d_{e1} + d_{e2}$ $\quad y_1 = y + d_{e3} + d_{e4}$

Note: Table values are based on $\lambda = 1.0$ and $f'_c = 5000$ psi;

for different material properties, multiply table values by $\lambda \sqrt{f'_c / 5000}$

ℓ_e in.	y_1, in. \ x_1, in.	2	4	6	8	10	12	14	16	18	20	22	24	26	28
		Design tensile strength, ϕP_{c1} (kips)													
3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2	1	1	2	3	3	4	5	5	6	7	7	8	9	9
	4	1	3	4	5	7	8	9	10	11	13	14	15	17	18
	6	2	4	6	8	9	11	13	15	17	19	21	23	25	27
	8	3	5	8	10	13	15	18	21	23	25	29	31	33	36
	10	3	7	9	13	16	19	23	25	29	32	35	39	42	45
	12	4	8	11	15	19	23	27	31	35	39	42	46	50	54
	14	5	9	13	18	23	27	31	36	41	45	49	54	59	63
	16	5	10	15	21	25	31	36	41	46	51	35	61	67	72
4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2	1	1	2	3	3	4	5	5	6	7	7	8	9	9
	4	1	3	4	5	7	8	9	10	11	13	14	15	17	18
	6	2	4	6	8	9	11	13	15	17	19	21	23	25	27
	8	3	5	8	10	13	15	18	21	23	25	28	31	33	36
	10	3	7	9	13	16	19	23	25	29	32	35	39	42	45
	12	4	8	11	15	19	23	27	31	35	39	42	46	50	54
	14	5	9	13	18	23	27	31	36	41	45	49	54	59	63
	16	5	10	15	21	25	31	36	41	46	51	57	61	67	72
6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2	1	1	2	3	3	4	5	5	6	7	7	8	9	9
	4	1	3	4	5	7	8	9	10	11	13	14	15	17	18
	6	2	4	6	8	9	11	13	15	17	19	21	23	25	27
	8	3	5	8	10	13	15	18	21	23	25	28	31	33	36
	10	3	7	9	13	16	19	23	25	29	32	35	39	42	45
	12	4	8	11	15	19	23	27	31	35	39	42	46	50	54
	14	5	9	13	18	23	27	31	36	41	45	49	54	59	63
	16	5	10	15	21	25	31	36	41	46	51	57	61	67	72
8	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2	1	1	2	3	3	4	5	5	6	7	7	8	9	9
	4	1	3	4	5	7	8	9	10	11	13	14	15	17	18
	6	2	4	6	8	9	11	13	15	17	19	21	23	25	27
	8	3	5	8	10	13	15	18	21	23	25	28	31	33	36
	10	3	7	9	13	16	19	23	25	29	32	35	39	42	45
	12	4	8	11	15	19	23	27	31	35	39	42	46	50	54
	14	5	9	13	18	23	27	31	36	41	45	49	54	59	63
	16	5	10	15	21	25	31	36	41	46	51	57	61	67	72